# Determination of vulnerability indexes for buildings and infrastructures designed to protect the territory of Ferrara from flood
# - Part b -

## Final Version

Deliverable Number D.3.3.2.

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|---|---|
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# INTRODUCTION

After the extensive damage provoked by the May 2012 seismic swarm on the pumping plants owned by the Burana and Central Emilia Land Reclamation Authority, the Ferrara Land Reclamation Authority has decided to apply seismic retrofitting measures on its pumping stations.

For this reason, it is necessary to assess the seismic vulnerability of such buildings in order to prioritize the interventions. In this deliverable, we summarize the outcomes of the seismic vulnerability analysis for three pumping stations owned by the Land Reclamation Authority of Ferrara: Valle Lepri Acque Alte, in the municipality of Comacchio, Sant'Antonino a Cona, in the municipality of Ferrara, and Guagnino in the municipaility of Comacchio.

The Valle Lepri pumping plant exhibits a very peculiar structural scheme, made of r.c. walls and very short r.c. columns, whose collapse behavior is typically due to shear failure and is fragile. The Sant'Antonino punping station covers a crucial role, in that it guarantees the hydraulic safety of the nearby Ferrara hospital. The Guagnino pumping plant serves a 73000ha area, mainly destined for agriculture, and its prevalent function is to lift exceedance waters at sea level.

# 1. VALLE LEPRI ACQUE ALTE

## 1.1 Description

The Valle Lepri pumping station is located in the municipality of Comacchio. Figure 1 depicts the location of the construction. The main building is a single storey structure with a rectangular footprint of dimensions 81.90 m × 14.80 m (see Figure 3). The interior space is almost entirely used for the drainage system, except for the end areas that are used as offices. The building was built in 1960 and consists of a reinforced concrete warehouse. The main structure is a frame consisting of 14 portals, spanning 14.90 m, connected to each other by means of perforated r.c. walls: the vertical loads are therefore beared by the columns while the walls enhance in-plane stiffness in the horizontal direction. The holes in the r.c. walls are covered by glass panes located at the base and at the top of the building: so that a series of very short columns is created. There is also a crane-holding beam that runs throughout the building, except the last bay (office area). The bridge-holding beam connects the entire shed also where the downpipes joints are located (Fig. 6). Along the shorter sides of the building there are masonry walls made of solid clay bricks, on which office floor slabs rest. The roof is made up of SAP 16 slabs resting on the beams at the top of the walls.

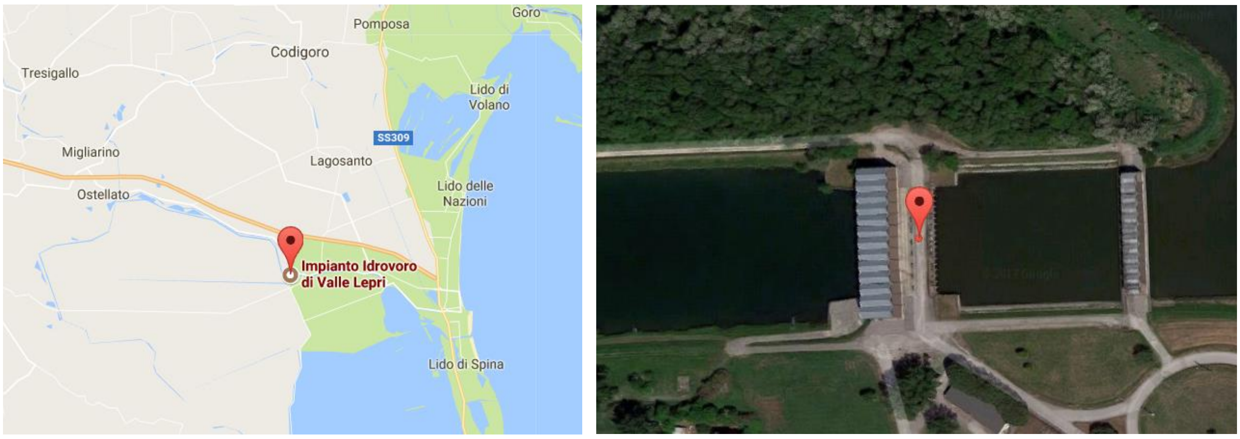



*Figure 1. Geographic localization of the Valle Lepri pumping plant.*

*Figure 2. Valle Lepri pumping plant.*

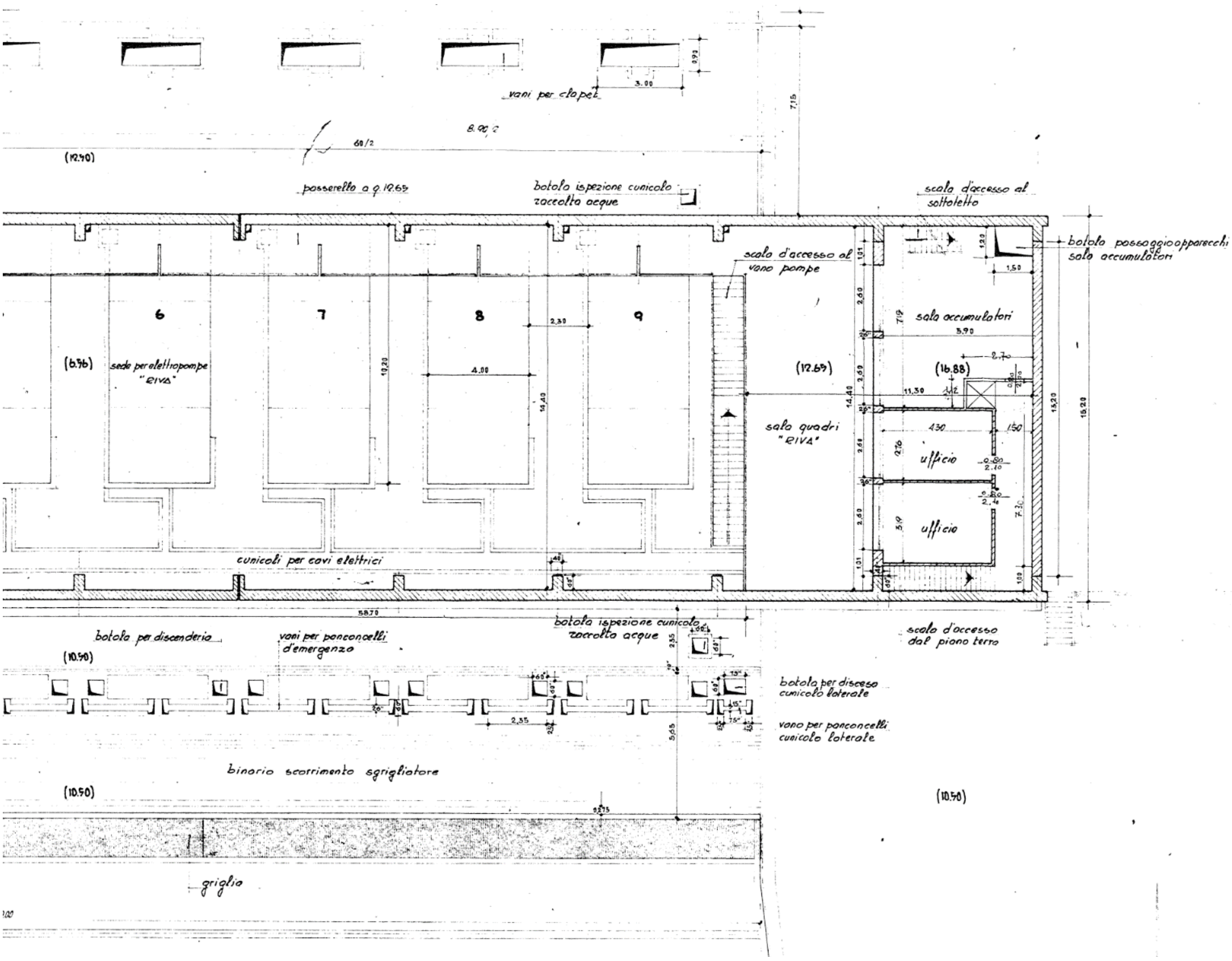



*Figure 3. Drawing of the ground floor.*

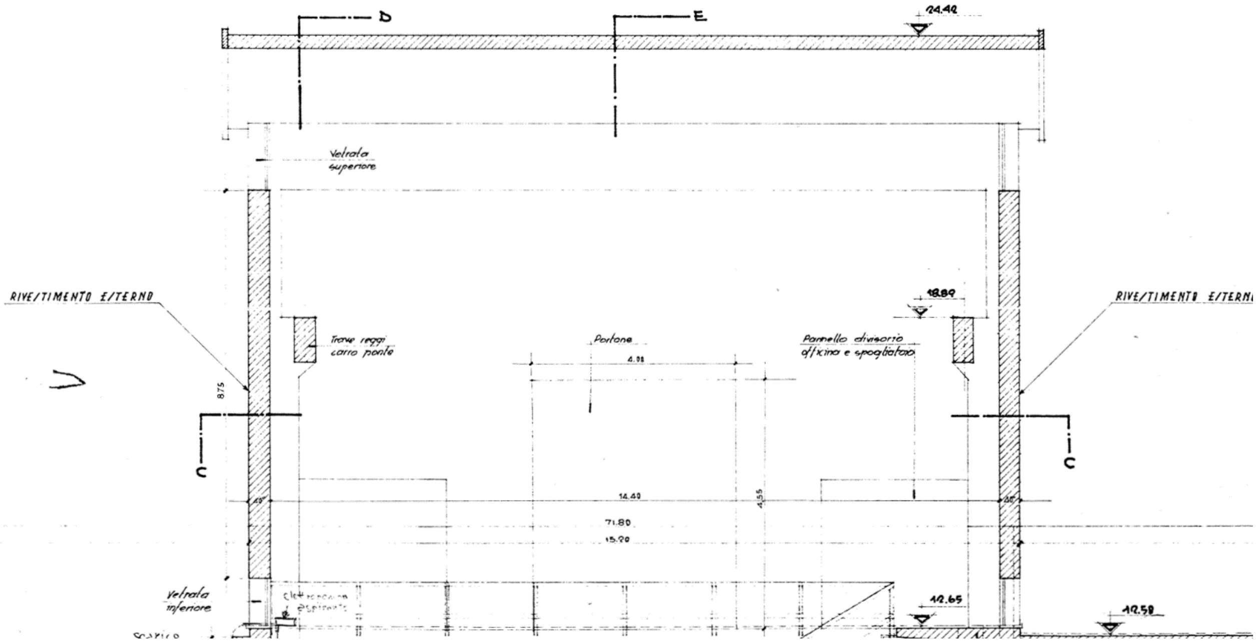



*Figure 4. Transverse section of the Valle Lepri pumping plant.*

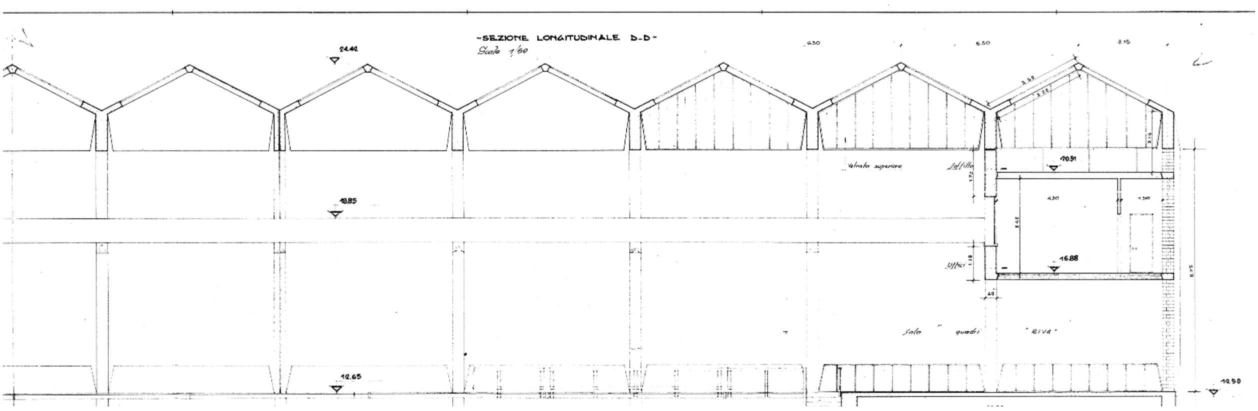



*Figure 5. Longitudinal section of the Valle Lepri pumping plant.*

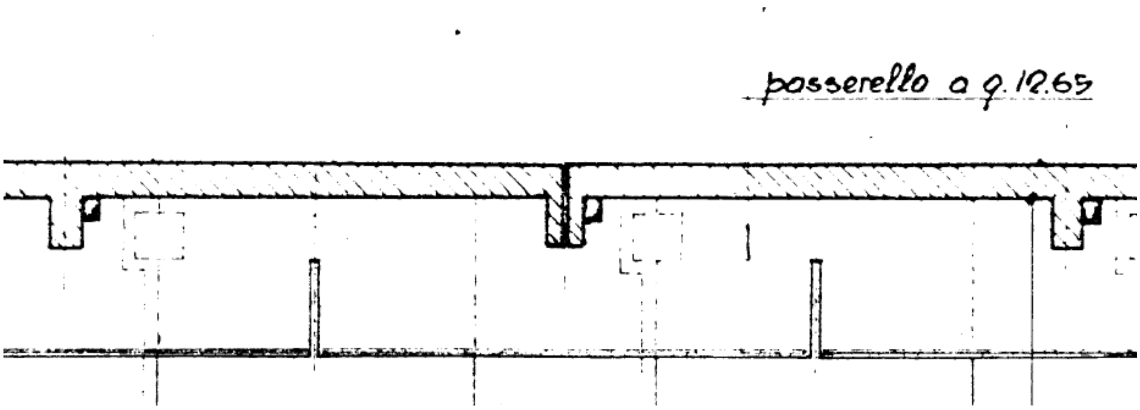



*Figura 6. Column sections.*

## 1.2 Material properties and loads

The structure is mainly subjected to the self weight of the structural elements:

- 25 $kN/m^3$ r.c. density;
- 18 $kN/m^3$ masonry density.

The floor slabs mainly undergo the follwing loads:

- Office floors:
  - Slab self weight: 3 $kN/m^2$
  - Non-structural loads: 1 $kN/m^2$
  - Accidental load: 2 $kN/m^2$
- Roof floor:
  - Slab SAP 16: 1.3 $kN/m^2$
  - Non-structural load: 1.70 $kN/m^2$
  - Snow load: not considered.

Regarding the parameters of the materials (ultimate strength of concrete and steel, masonry elasticity modules), at this preliminary stage, ordinary values have been adopted: concrete C25 / 30 and FeB22k steel (yield strength fyd = 187 MPa ).

## 1.3 Design seismic action

The design seismic action is defined through the horizontal psuedo-acceleration spectrum as per the current Italian Building Code presriptions.

- Nominal life: VN=50 years.
- Usage coefficient: Cu=1.5 .
- Limit State: SLV
- Soil category: D
- Topographic category: T1
- Behavior factor: q=1.5.

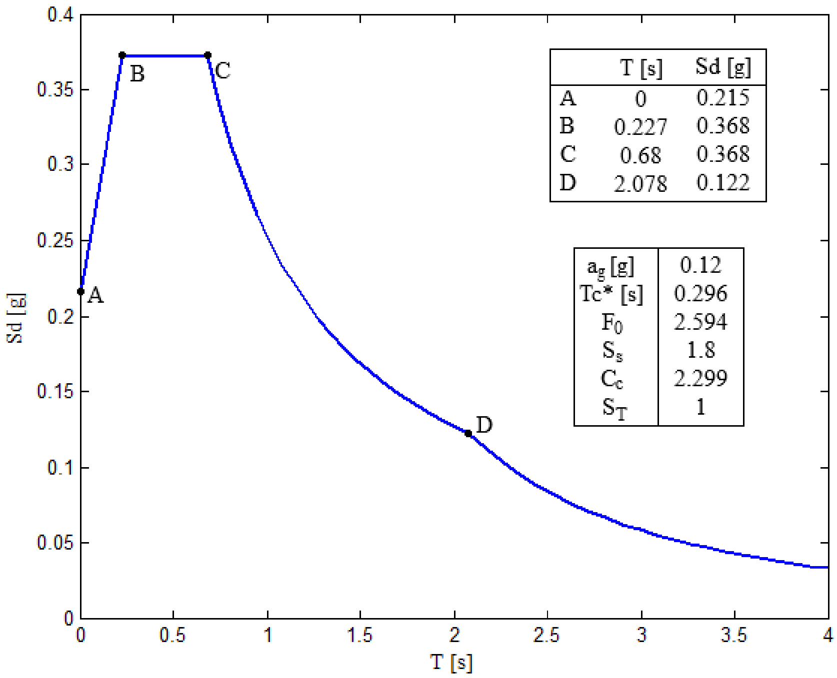



*Figura 7. Spettro di risposta orizzontale adottato.*

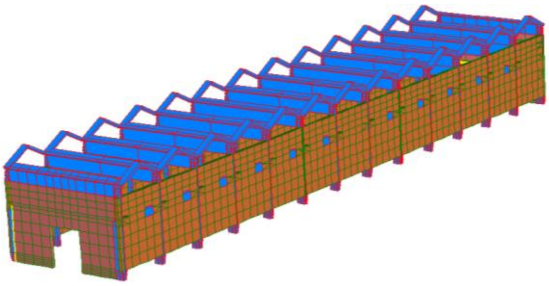
# 1.4 Analyses performed

The following analyzes were performed by means of FEM models using the Straus7 and ProSap software (Figure 8). For both models, linear static analyses with uniform acceleration and modal analyses have been developed. Here the reference system adopted is the following:

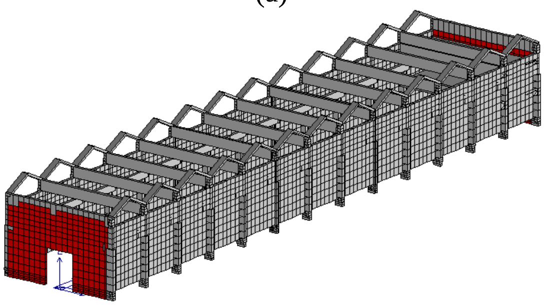
- X direction: axis directed parallel to the shorter side of the building;
- Y direction: axis directed parallel to the long side of the building;
- Z direction: vertical axis.

The seismic mass has been modeled according to the seismic combinations specified in paragraph 2.5.3. of the NTC2008. Subsequently the earthquake was applied with the following 8 combinations: $\pm Ex \pm 0.3Ey$

– $\pm 0.3Ex \pm Ey$

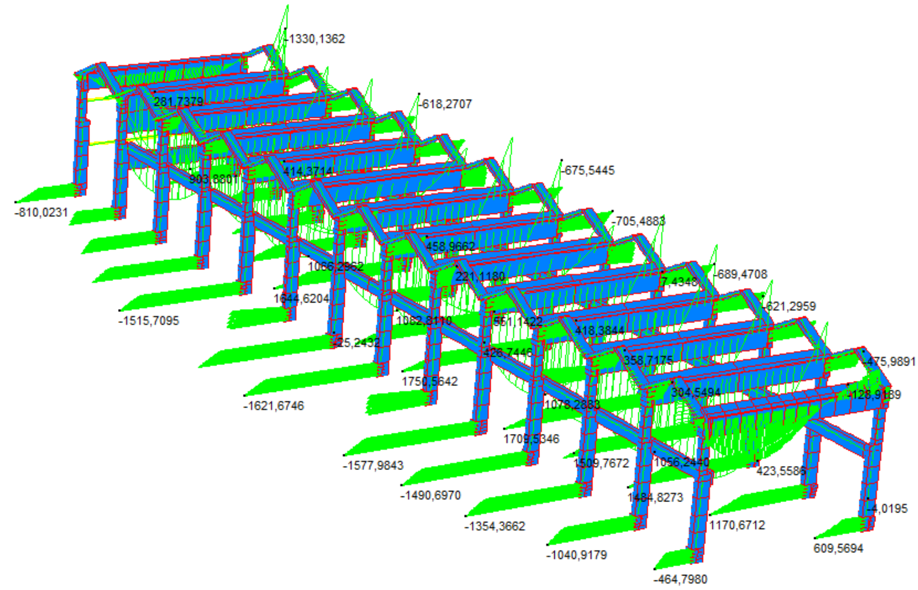


(a)

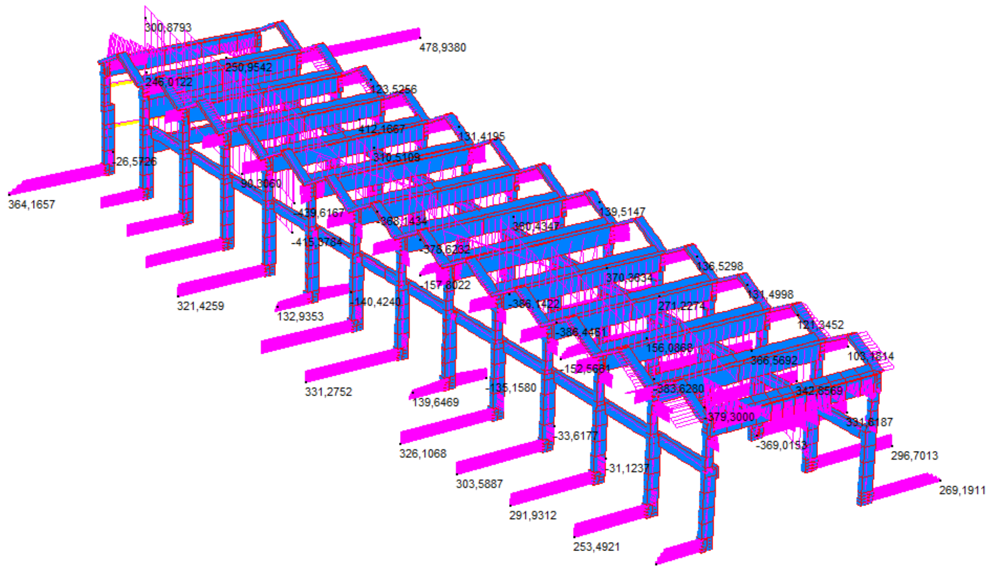


(b)

*Figure 8. FEM models: (a) Straus7, (b) ProSap.*

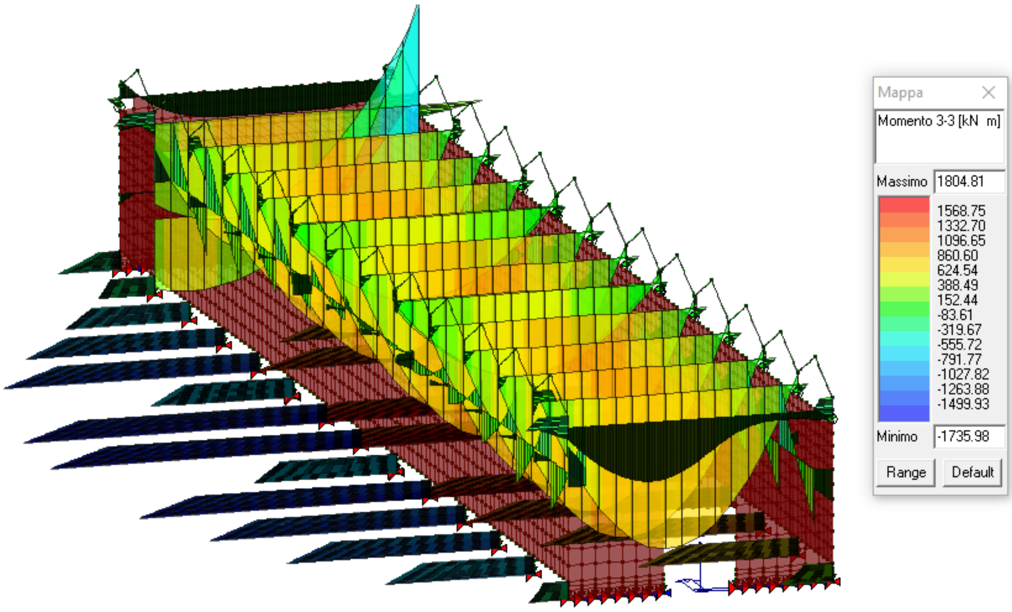



(a)

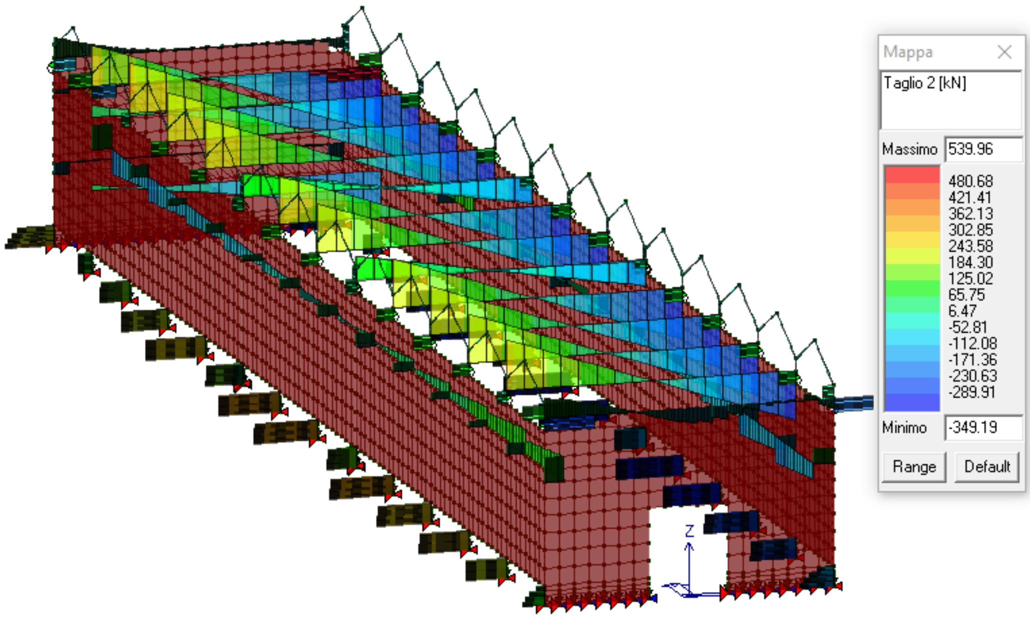



(b)

*Figure 9. Internal actions computed in Straus7 for the Ex+0.3Ey load combination: (a) bending moment envelope (b) shear forces.*




(a)




(b)

*Figure 10. Internal actions computed in ProSAP for the Ex+0.3Ey load combination: (a) bending moment envelope (b) shear forces.*

## 1.5 Shear capacity assessment of the short columns

The shear capacity of the r.c. columns is given by

$$V_{Rd} = 0.9 \cdot d \cdot \frac{A_{sw}}{s} \cdot f_{yd} \cdot (\cot\alpha + \cot\theta) \cdot \sin\alpha$$

where:

$d$ : section depth;

$\frac{A_{sw}}{s}$ : shear reinforcement area;

$f_{yd}$ : steel design yield strength;

$\alpha$ : tilt angle of the shear reinforcement;

$\theta$ : tilt angle of the r.c. struts.

The formula refers to slender columns (therefore characterized by Euler-Bernoulli behavior). In the case of squat columns the calculation of the shear capacity has been modified by inserting in the formula the exact value of the lever arm of the internal couple, in order to take into account the actual position of the concrete strut. The shear capacity is therefore evaluated as follows:

$$V_{Rd} = \left(d - \frac{x}{2}\right) \cdot \frac{A_{sw}}{s} \cdot f_{yd} \cdot (\cot\alpha + \cot\theta) \cdot \sin\alpha$$

The lever arm of the internal couple can therefore be expressed as:

$$b = \min\left(0.9 \cdot d\,;\ d - \frac{x}{2}\right)$$

The following table summarizes the values of the neutral axis position and the lever arm of the internal couple used for the shear assessment of the short columns.

| **Column section** | **Neutral axis position (x) and lever arm of the internal couple (b)** | | | |
|---|---|---|---|---|
| | X direction | | Y direction | |
| | Medx≥Mrdx | Medx<Mrdx | Medy≥Mrdy | Medy<Mrdy |
| **40×100** | x=0.2d ; b=0.9d | x=0.5d ; b=0.5d | x=0.2d ; b=0.9d | x=0.5d ; b=0.5d |
| **19×100** | x=0.3d ; b=0.85d | x=0.5d ; b=0.5d | x=0.3d ; b=0.85d | x=0.5d ; b=0.5d |

*Table 1. Values of the neutral axis position and the lever arm of the internal couple used for the shear assessment of the short columns.*

## 1.6 Results

In the following, we report the structural assessment of the short columns undergoing seismic horizontal loads: for each seismic combination, the internal actions acting on the most stressed column have been reported.

The bending moment capacity has been obtained by means of the VcaSlu software. The results obtained highlight a strong shear vulnerability on the short columns especially in the Y direction. The corresponding safety factor is 8% (section 40 × 100).

### 1.6.1 Column section 40×100

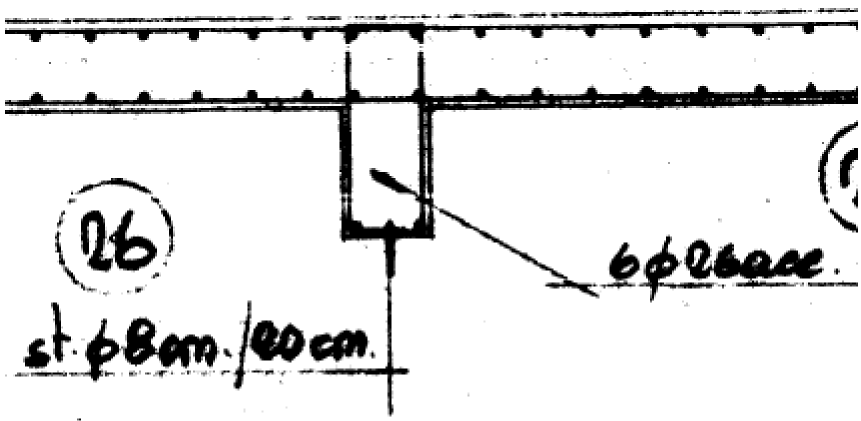



*Figure 11. Steel reinforcement in 40×100 columns.*

| | | |
|---|---|---|
| **Shear reinforcement** | Φ | 8 |
| | bracci | 2 |
| | passo [cm] | 20 |
| | As/s [mm2/mm] | 0.50 |

| | | |
|---|---|---|
| **Shear reinforcement** | Φ | 8 |
| | bracci | 4 |
| | passo [cm] | 20 |
| | As/s [mm2/mm] | 1.01 |

| | | |
|---|---|---|
| | fyd [MPa] | 187 |
| R.C. struts | x [d] | 0.2 |
| | d [mm] | 970 |
| | braccio coppia interna [mm] | 873 |
| | θ [°] | 45 |
| **Vrdx [kN]** | 82.1 | |

| | | |
|---|---|---|
| | fyd [MPa] | 187 |
| R.C. struts | x [d] | 0.2 |
| | d [mm] | 370 |
| | braccio coppia interna [mm] | 333 |
| | θ [°] | 45 |
| **Vrdy [kN]** | 62.6 | |

*Table 2. Shear capacity in the X and Y directions.*

The following table reports the bending and shear assessment for different load combinations:

| Load combination | N [kN] | Mx [kNm] | My [kNm] | Mrdx [kNm] | Mrdy [kNm] | 0.7Mrdx/ Mx | 0.7Mrdy/ My |
|---|---|---|---|---|---|---|---|
| Ex+0.3Ey | 887.1 | 64 | 1804.81 | 290.6 | 637.9 | 3.18 | 0.25 |
| Ex-0.3Ey | 945 | 33.93 | 1804.57 | 297.3 | 652.8 | 6.13 | 0.25 |
| -Ex+0.3Ey | 887.1 | 64 | 1804.81 | 290.6 | 637.9 | 3.18 | 0.25 |
| -Ex-0.3Ey | 945 | 33.93 | 1804.57 | 297.3 | 652.8 | 6.13 | 0.25 |
| 0.3Ex+Ey | 758.68 | 225.07 | 373.82 | 286 | 625.7 | 0.89 | 1.17 |
| -0.3Ex+Ey | 758.68 | 225.07 | 373.82 | 286 | 625.7 | 0.89 | 1.17 |
| 0.3Ex-Ey | 874 | 219.2 | 191 | 288.4 | 633.1 | 0.92 | 2.32 |
| -0.3Ex-Ey | 874 | 219.2 | 191 | 288.4 | 633.1 | 0.92 | 2.32 |

*Table 3. Bending assessment for columns with section 40×100.*

| Load combination | Vx max [kN] | Vy max [kN] | Vrdx [kN] | Vrdy [kN] | Vrdx/Vedx | Vrdy/Vedy |
|---|---|---|---|---|---|---|
| Ex+0.3Ey | 325.83 | 451.36 | 82.1 | 34.78 | 0.25 | 0.08 |
| Ex-0.3Ey | 325.95 | 427.63 | 82.1 | 34.78 | 0.25 | 0.08 |

| | | | | | | |
|---|---|---|---|---|---|---|
| -Ex+0.3Ey | 325.83 | 451.36 | 82.1 | 34.78 | 0.25 | 0.08 |
| -Ex-0.3Ey | 325.95 | 427.63 | 82.1 | 34.78 | 0.25 | 0.08 |
| 0.3Ex+Ey | 113.08 | 506.32 | 45.6 | 62.60 | 0.40 | 0.12 |
| -0.3Ex+Ey | 113.08 | 506.32 | 45.6 | 62.60 | 0.40 | 0.12 |
| 0.3Ex-Ey | 112.6 | 493.38 | 45.6 | 62.60 | 0.40 | 0.13 |
| -0.3Ex-Ey | 112.6 | 493.38 | 45.6 | 62.60 | 0.40 | 0.13 |

*Table 4. Shear assessment for columns with section 40×100.*

## 1.6.2 Column section 19×100

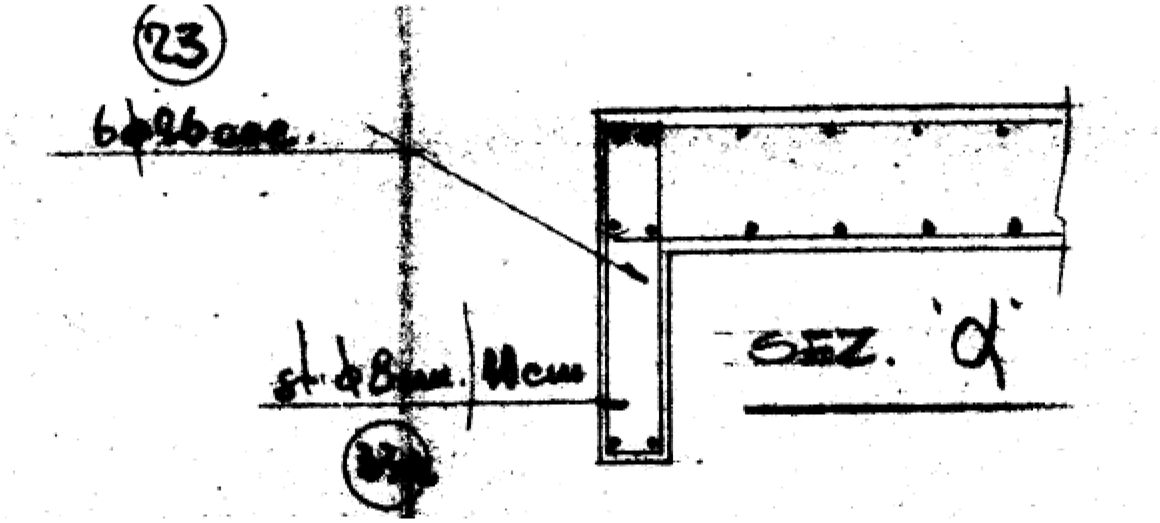



*Figura 12. Steel reinforcement*

| | | |
|---|---|---|
| **Shear reinforcement** | Φ | 8 |
| | bracci | 2 |
| | passo [cm] | 20 |
| | As/s [mm2/mm] | 0.50 |

| | | |
|---|---|---|
| **Shear reinforcement** | Φ | 8 |
| | bracci | 4 |
| | passo [cm] | 20 |
| | As/s [mm2/mm] | 1.01 |

| | | |
|---|---|---|
| | fyd [MPa] | 187 |
| **R.C. struts** | x [d] | 0.2 |
| | d [mm] | 970 |
| | braccio coppia interna [mm] | 873 |
| | θ [°] | 45 |
| **Vrdx [kN]** | 82.1 | |

| | | |
|---|---|---|
| | fyd [MPa] | 187 |
| **R.C. struts** | x [d] | 0.2 |
| | d [mm] | 160 |
| | braccio coppia interna [mm] | 144 |
| | θ [°] | 45 |
| **Vrdy [kN]** | 27.1 | |

*Table 5. Shear capacity in the X and Y directions.*

The following table reports the bending and shear assessment for different load combinations:

| **Combinazione** | **N [kN]** | **Mx [kNm]** | **My [kNm]** | **Mrdx [kNm]** | **Mrdy [kNm]** | **0.7Mrdx/ Mx** | **0.7Mrdy/ My** |
|---|---|---|---|---|---|---|---|
| Ex+0.3Ey | 316.82 | 9.97 | 823.89 | 65.79 | 369.1 | 4.62 | 0.31 |
| Ex-0.3Ey | 219.28 | 10 | 824.55 | 60.38 | 342.8 | 4.23 | 0.29 |
| -Ex+0.3Ey | 316.82 | 9.97 | 823.89 | 65.79 | 369.1 | 4.62 | 0.31 |
| -Ex-0.3Ey | 219.28 | 10 | 824.55 | 60.38 | 342.8 | 4.23 | 0.29 |
| 0.3Ex+Ey | 537 | 53.54 | 228.41 | 77.68 | 414.6 | 1.02 | 1.27 |
| -0.3Ex+Ey | 537 | 53.54 | 228.41 | 77.68 | 414.6 | 1.02 | 1.27 |
| 0.3Ex-Ey | 565.95 | 58.39 | 226.49 | 76.06 | 418.9 | 0.91 | 1.29 |
| -0.3Ex-Ey | 565.95 | 58.39 | 226.49 | 76.06 | 418.9 | 0.91 | 1.29 |

*Table 6. Bending assessment for columns with section 19×100.*

| **Combinazione** | **Vx max [kN]** | **Vy max [kN]** | **Vrdx [kN]** | **Vrdy [kN]** | **Vrdx/Vedx** | **Vrdy/Vedy** |
|---|---|---|---|---|---|---|
| Ex+0.3Ey | 194.16 | 116.87 | 77.5 | 15.04 | 0.40 | 0.13 |
| Ex-0.3Ey | 193.68 | 150.33 | 77.5 | 15.04 | 0.40 | 0.10 |

| | | | | | | |
|---|---|---|---|---|---|---|
| -Ex+0.3Ey | 194.16 | 116.87 | 77.5 | 15.04 | 0.40 | 0.13 |
| -Ex-0.3Ey | 193.68 | 150.33 | 77.5 | 15.04 | 0.40 | 0.10 |
| 0.3Ex+Ey | 62.38 | 149.69 | 45.6 | 25.57 | 0.73 | 0.17 |
| -0.3Ex+Ey | 62.38 | 149.69 | 45.6 | 25.57 | 0.73 | 0.17 |
| 0.3Ex-Ey | 60.76 | 162.57 | 45.6 | 25.57 | 0.75 | 0.16 |
| -0.3Ex-Ey | 60.76 | 162.57 | 45.6 | 25.57 | 0.75 | 0.16 |

*Table 7. Shear assessment for columns with section 19×100.1.7 Conclusioni*

As expected, the most vulnerable elements are the short columns at the base, on which the highest values of shear and moment stresses are concentrated. In addition to the lack of slenderness, they have a variable section that gets thinner as it approaches the height of the seismic zero: the section is therefore weaker where there are the greatest shear and bending moment stresses.

With the assumptions adopted in this model, the safety level appears to be 8%.
A first proposal, obviously not feasible, of intervention is to close the openings in the partitions at the base by casting new concrete elements collaborating with the existing columns. This would eliminate the weak sections, allowing the distribution of the total shear force on all the walls instead of only on the pillars.

# 2. S. ANTONINO

## 2.1 Description

The Sant'Antonino pumping station is located in Cona (Fe). Figure 13 shows the location and aerial photography of the building. The structure that houses the plant is a group of several adjacent masonry buildings (Figure 15, Figure 16). The lateral body exposed to the East is currently used for maintenance of the plant, while the lateral body exposed to the West is used as an archive. It has not been possible to trace the quality of the clamping between the longitudinal walls constituting the central body and the transverse walls constituting the perimeter walls of the lateral bodies adjacent to the central body. At present, however, there is some plaster damage that seem to indicate a poor fading between the various bodies, or a joint of inadequate dimensions. The horizontal dimensions of the central body are 18 m x 9.40 m, with a height of 9.10 m. The two main walls are 18 m long and have a thickness that decreases with height: 44 cm up to the height of 5.35 m where there is the bridge crane, and 28 cm above it. This wall supports r.c. beams (section 20 × 35 cm) on which rests a slab with a total height of 16 cm. The roofing is of the two-pitch type with the ceiling assumed to be of the same type as the floor slab. The lateral bodies have horizontal dimensions of approximately 10 × 12 m for a height of 11 m. They consist of 30 cm solid clay masonry. Both side bodies have the same total height and inter-floor. Since it was not possible to carry out more in-depth and invasive investigations, the first floor and the under-roof were supposed to have a height of 16 cm with a 4 cm concrete slab. The presence of r.c. curbs has also been assumed.


*Figure 13. Geographic localization of the S. Antonino pumping plant.*

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*Figure 14. S. Antonino pumping station.*

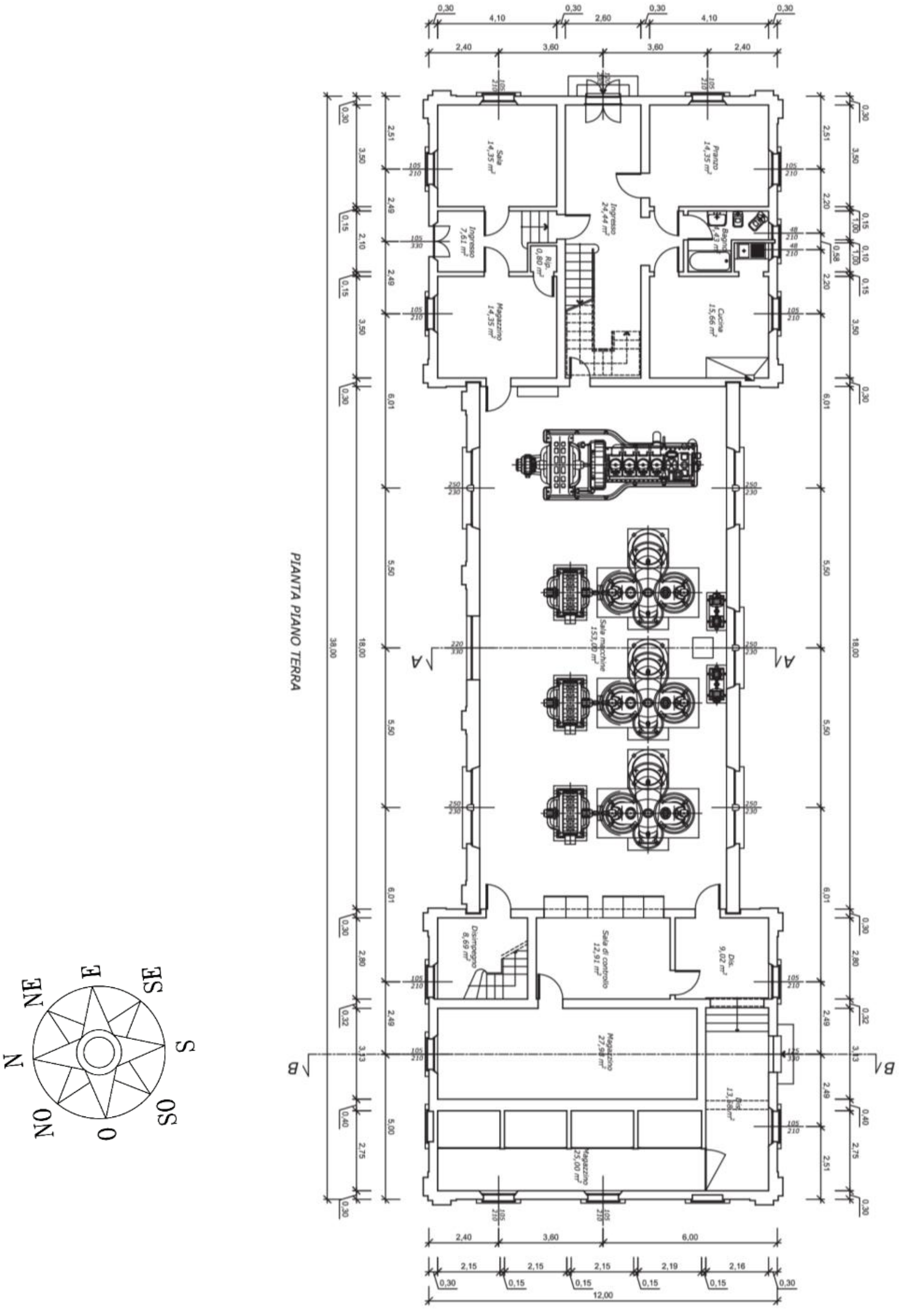


*Figure 15. S. Antonino pumping station horizontal section, ground floor.*

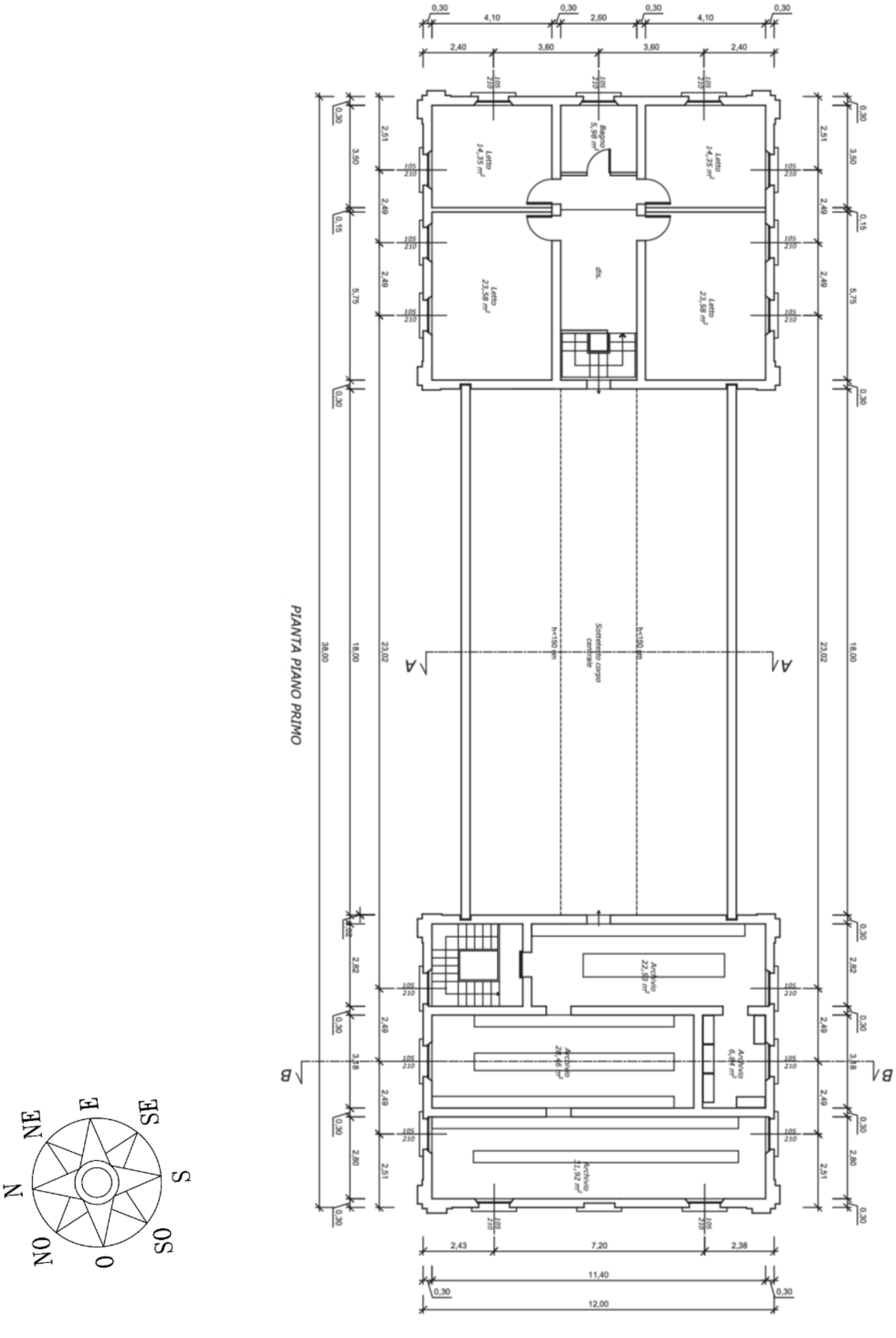


*Figure 16. S. Antonino pumping station horizontal section, first floor.*

## 2.2 Material properties

The buildings consist of solid clay masonry and lime mortar with good characteristics. Only for the two lateral bodies a good connection between the orthogonal walls is assumed. The material improvement coefficients prescribed by the Circular were therefore applied to the masonry parameters (Table C8A.2.2):

- "Good quality mortar": 1.5;
- "Good connection between walls": 1.3.

| | |
|---|---|
| Average compression strength | $f_m = 1.5 \cdot 1.3 \cdot 240 = 468\ [N/cm^2]$ |
| Average shear strength | $\tau_m = 1.5 \cdot 1.3 \cdot 6 = 11.7\ [N/cm^2]$ |
| Average Young modulus | $E = 1.5 \cdot 1500 = 2250\ [N/mm^2]$ |
| Average shear modulus | $G = 1.5 \cdot 500 = 750\ [N/mm^2]$ |
| Specific weight | $w = 18\ [kN/m^3]$ |

*Table 8. Adopted mechanical parameters for masonry.*

## 2.3 Loads

| **Ceiling (Central body)** | | | |
|---|---|---|---|
| **Material** | **□□□kN/m$^3$]** | **t [cm]** | **Weight[kN/m$^2$]** |
| Plaster | 20 | 1 | 0.20 |
| R.C. beams (20x35) int. 2.2 m | - | - | 1.00 |
| 16cm slab | - | - | 1.85 |
| **Totale** | | | **3.05** |
| **Peso permanente strutturale $G_1$** | | | **2.85** |
| **Peso permanente non strutturale $G_2$** | | | **0.20** |
| **Carico accidentale (sottotetto) $Q_{k1}$** | | | **0.50** |
| **Roof (Central body)** | | | |

| | □□□kN/m$^3$] | t [cm] | Weight [kN/m$^2$] |
|---|---|---|---|
| 16 cm slab | - | - | 1.85 |
| Impermeabilization | - | - | 0.10 |
| Tiles | - | - | 0.45 |
| **Total** | | | **2.40** |
| **Structural weight $G_1$** | | | **1.85** |
| **Non structural weight $G_2$** | | | **0.55** |
| **Accidental load (snow) $Q_{k1}$** | | | **0.80** |

*Table 9. Loads on the central body.*

| **First floor (Lateral bodies)** | | | |
|---|---|---|---|
| **Material** | □□□kN/m$^3$] | t [cm] | Weight [kN/m$^2$] |
| Plaster | 20 | 1 | 0.20 |
| 16+4 slab | - | - | 2.85 |
| Light concrete | 17 | 3 | 0.51 |
| Pavement | - | - | 0.40 |
| Internal divisions | - | - | 1.00 |
| **Total** | | | **4.96** |
| **Structural weight $G_1$** | | | **2.85** |
| **Non structural weight $G_2$** | | | **2.11** |
| **Accidental load 1 $Q_{k1}$** | | | **2.00** |
| **Accidental load 2 $Q_{k1}$** | | | **6.00** |
| **Second floor (Lateral bodies)** | | | |

| *Material* | □□□kN/m$^3$] | t [cm] | Weight [kN/m$^2$] |
|---|---|---|---|
| Plaster | 20 | 1 | 0.20 |
| 16+4 slab | - | - | 2.85 |
| **Total** | | | **3.05** |
| **Structural weight $G_1$** | | | **2.85** |
| **Non structural weight $G_2$** | | | **0.20** |
| **Accidental load $Q_{k1}$** | | | **0.50** |
| **Roof (Lateral bodies)** | | | |
| | □□□kN/m$^3$] | t [cm] | Weight [kN/m$^2$] |
| 16+4 slab | - | - | 2.85 |
| Impermeabilization | - | - | 0.10 |
| Tiles | - | - | 0.45 |
| **Total** | | | **3.40** |
| **Structural weight $G_1$** | | | **2.85** |
| **Non structural weight $G_2$** | | | **0.55** |
| **Accidental load $Q_{k1}$** | | | **0.80** |

*Table 10. Loads on lateral bodies.*

## 2.4 Design seismic action

The design seismic action is defined through the horizontal psuedo-acceleration spectrum as per the current Italian Building Code presriptions.

– Nominal life: VN=50 years.
– Usage coefficient: Cu=1.5 .
– Limit State: SLV
– Soil category: D

- Topographic category: T1
- Behavior factor: q=1.

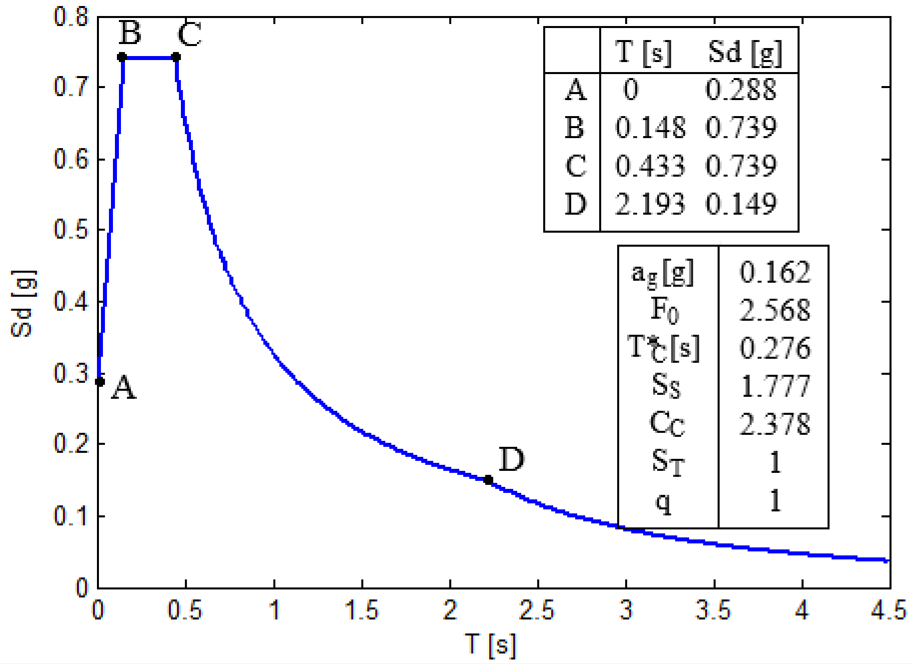



*Figure 17. Psuedo-acceleration design spectrum.*

## 2.5 Structural analyses

We assume poor connection between the walls of the central body and the lateral buildings. Therefore, the three buildings are assumed to be completely disconnected and studied separately. First, preliminary analyses were carried out on the structure undergoing vertical loads only (fundamental combination, specified in paragraph 2.5.3 of the NTC2008). Subsequently, the buildings' response to horizontal loads was analyzed.

With regard to the central body, consisting of only two parallel walls that support the roof, only the out-of-plane local mechanisms have been assessed using both the Mc4Loc software and manual calculations. On the lateral buildings, on the other hand, given the hypothesis of good overall behavior, non-linear global static (pushover) analyses were developed using the 3Muri software, The seismic masses were modeled according to the seismic combination specified in paragraph 2.5.3 of the NTC2008. Subsequently the earthquake was applied with the following 8 combinations:

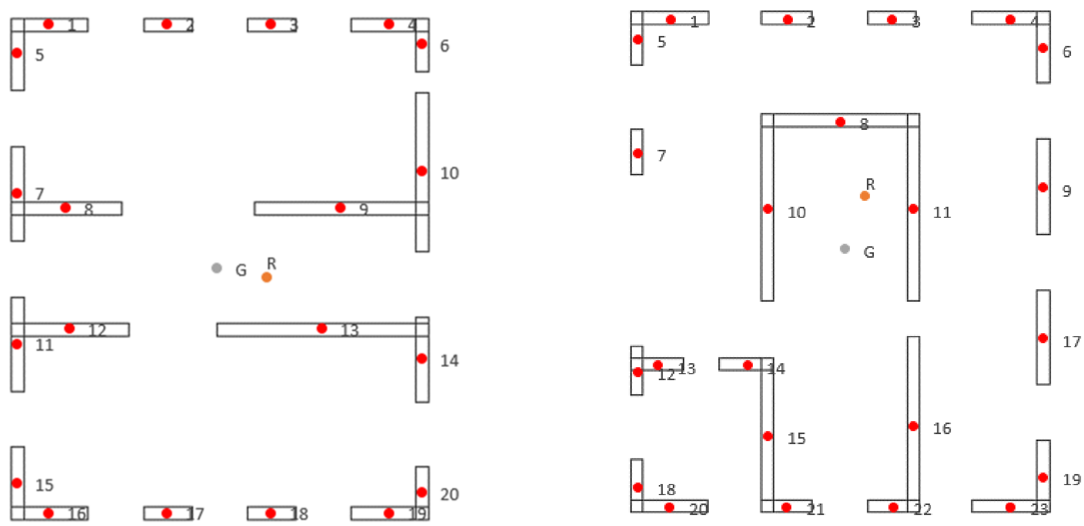
- $\pm Ex \pm 0.3Ey$
- $\pm 0.3Ex \pm Ey$

# 2.6 Results

## 2.6.1 Vertical loads

The walls making up the central body are characterized by a non-uniform thickness along its height. Therefore, it was decided to adopt the thickness to be used for the slenderness calculation using a weighted average (on the heights). Through this procedure a calculation slenderness of 16 was obtained. With this assumption, however, the walls of the central body are not verified to vertical loads (minimum safety index of 78.7%). However, more information on the properties of the materials are needed (it was observed that using a unitary confidence factor these walls would be verified).

In the lateral bodies, some masonry walls are unverified to vertical loads. This can be attributed both to the high eccentricity of the loads and to the factorization used. The verification was limited to the walls of the ground floor as they are more loaded.




(a) (b)

*Figure 18. (a): left building, non verified walls: 1,2,3,4,16,17,18,19; (b): right building, non-verified walls: 6,7,9,12,15,17,19.*

## 2.6.2 Local mechanisms in the central building

The following mechanisms have been analyzed:

- Complete overturning of the walls;
- Partial overturning of the walls;
- Expulsion.

Table 12 shows the lower safety coefficients (capacity / demand) obtained from local anlyses. The worst situation was found with reference to the complete overturning of the walls, for which there is a capacity / demand ratio corresponding to 19.66% (linear kinematic analysis procedure).

## 2.6.3 Global analyses on lateral buildings

In order to assess the seismic vulnerability of the plant a 3Muri model of each building is realized. In each model all the walls do not comply with the geometric requirements (Tab. 7.8.III DM08) and slenderness ($\lambda \leq 12$) imposed by the standard in chap. 7 of the DM08. The 3Muri models are shown below.

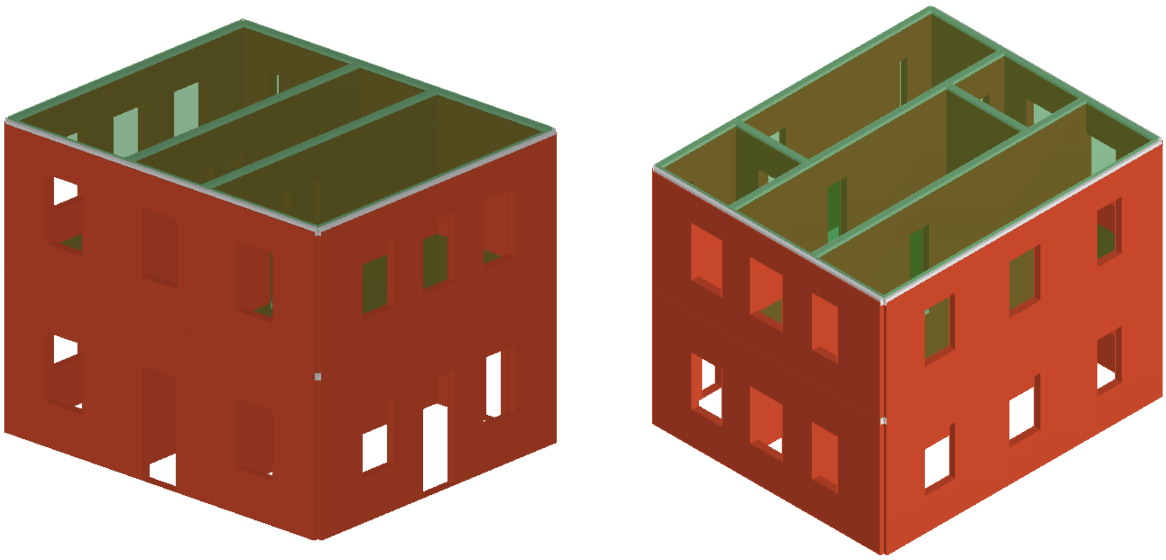


*Figure 19. 3Muri models of the two lateral buildings.*

The models were analyzed with both the hypotheses of presence and absence of reinforced concrete curbs. The computed safety factors are shown in Table 12.

The analyses highlight the presence of a "weak direction" for both buildings, ie a direction in which there is not enough masonry to withstand the seismic action. This result is also validated by a verification carried out according to the limits reported in Table 7.8.III of NTC2008.

### 2.6.4 Summary of the analyses with horizontal loads

The following is a summary of the results obtained regarding the seismic vulnerability analysis of the buildings. In addition to the safety factors (column αRd / αEd) the return period (column TI) was also reported.

| | **Description** | $a_{Rd}$ $[m/s^2]$ | $a_{Ed}$ $[m/s^2]$ | $\frac{a_{Rd}}{a_{Ed}}$ | $TR_{Rd}$ $[anni]$ | **TI** $[anni]$ |
|---|---|---|---|---|---|---|
| **Local mechanisms (Central building)** | Overturning | 0.283 | 1.439 | 19.7 % | 30 | 2 |
| | Partial overturning | 0.920 | 2.954 | 31.1 % | 57 | 4 |
| | Expulsion | 3.928 | 1.439 | 273 % | > 712 | > 50 |
| **Global analysis (Lateral buildings)** | Left building | 1.39 | 2.85 | 48.8 % | 136 | 10 |
| | Right building | 1.15 | 2.85 | 40.4 % | 93 | 7 |

*Table 11. Summary of the verifications w.r.t. horizontal loads.*

## 2.7 Proposed interventions

### 2.7.1 Steel tie-rods

As can be seen from the calculations, the verifications against simple overturning of the entire façade are not satisfied. Therefore, to improve the existing safety level with regard to the seismic action, it is suggested to insert tie-rods at the level of the intrados of the beams carrying the attic floor.

In order to carry out the necessary verifications, S275 steel tie-rods with a diameter of mm 20 mm, on a 50x50 cm square plate have been adopted.

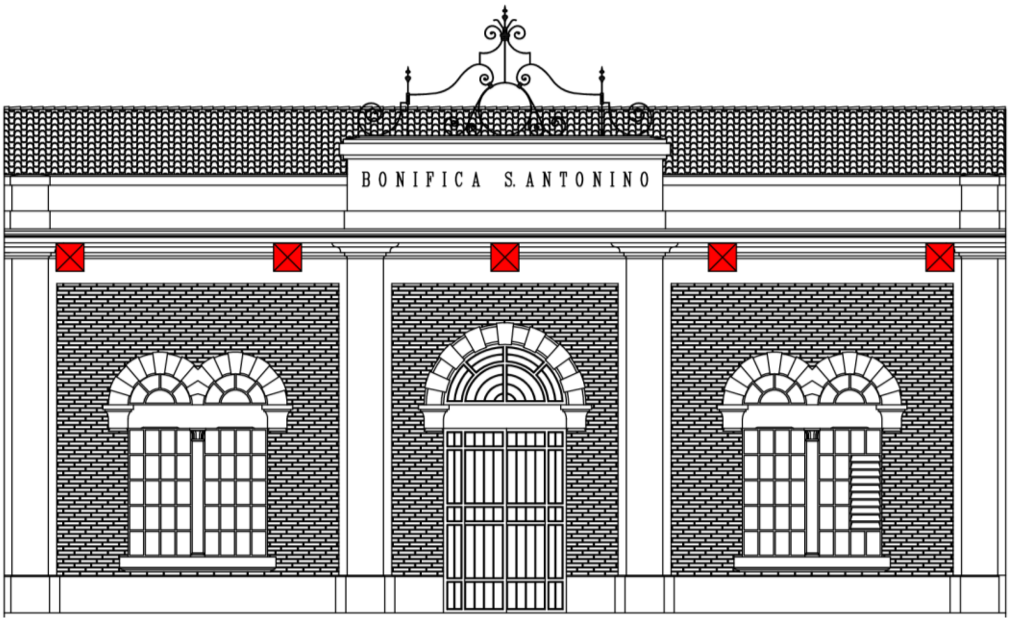



*Figura 20. Disposizione dei tiranti sul prospetto Nord.*

## 2.7.2 Overturning restraining system

In order to remedy the partial (top) overturning of the walls on the North and South side, it is possible to install an overturning restraining system anchored to the wall and to the side buildings that prevent the occurrence of this mechanism.

For this purpose, it is possible to prevent overturning by installing a UPN120 type metal profile at the upper edge of this wall using bolts fixed with chemical anchors. At the end of this profile it was decided to weld an L-shaped plate 30x30x82 cm with a thickness of 10 mm and to continue on the wall of the adjacent building by welding an additional UPN profile to the plate for another 125 cm. The use of this plate was necessary in order to be able to install an adequate number of bolts, since the weak point of the connections with the use of chemical anchors in the masonry is the low shear strength declared by the manufacturers of these anchors.

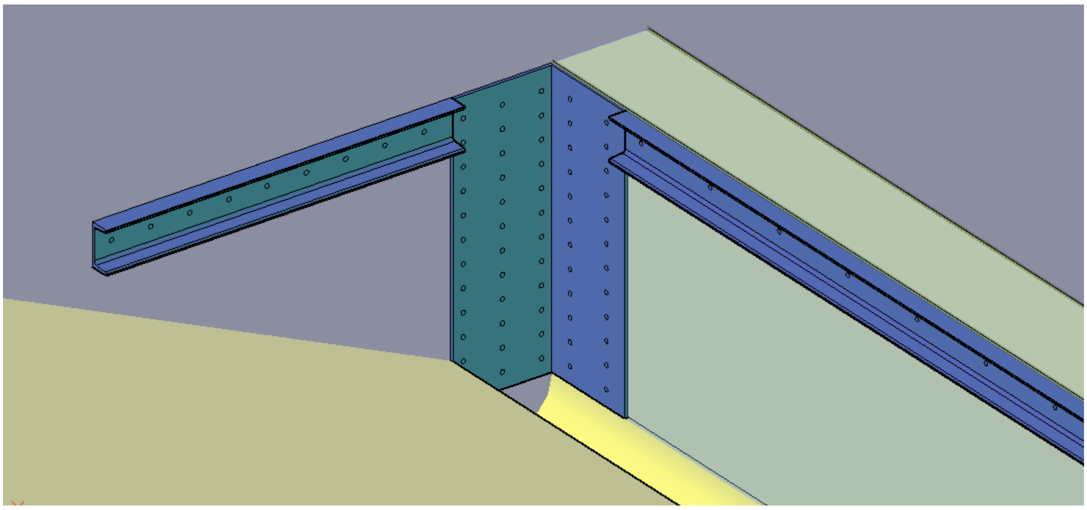


*Figure 21. 3D particular of the proposed overturning restraining system.*

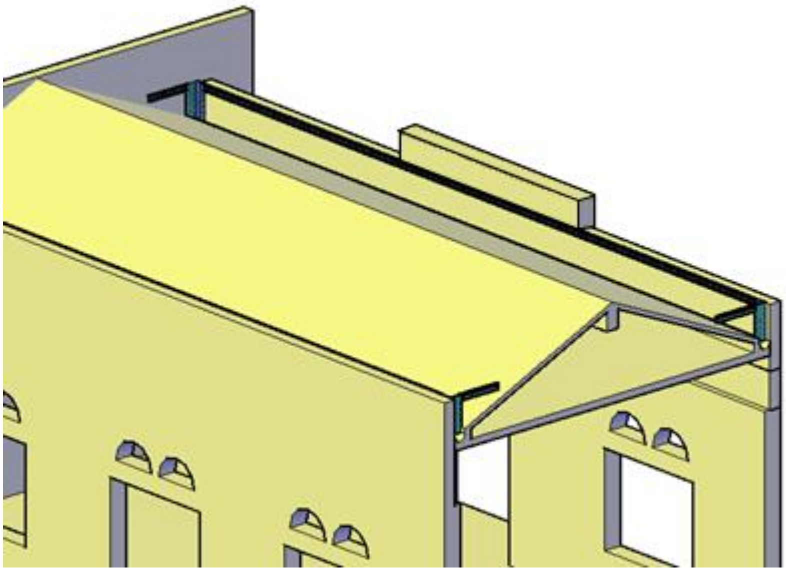


*Figure 22. 3D global scheme of the proposed overturning system.*

## 2.8 Conclusions

The analyses carried out have shown vulnerability to vertical loads for all buildings and strong vulnerabilities to horizontal loads for the central body.

As for the side buildings, a possible solution is to change the intended use to the building on the left: the idea is to use the building only for use as a "storage room" or an archive, while maintaining the same operating loads as the civil dwelling. The goal of this operation is to lower the probability of having people inside the plant in the event of an earthquake.

The worst vulnerabilities occur in any case with horizontal loads, as can be seen in Table 12. In particular, the worst deficiencies have been highlighted by local analyzes on the walls of the central body. As a matter of fact, therefore, there is a security index of 19.7% (reversal of the North wall), which corresponds to an intervention time of 2 years.

The proposed interventions are aimed at averting the most serious crises identified, that is to say the total and partial reversals of the walls of the central body. The application of tie rods can prevent the tilting mechanism, while the metal curb prevents the walls from falling down at high altitude. Once these mechanisms are prevented, the subsequent vulnerabilities are found on the global analyzes of the side buildings. The application of the proposed interventions would therefore bring the safety index of the structure to 40.4% (global response to the earthquake of the building on the right) and the time of intervention to 7 years.

# 3. GUAGNINO

## 3.1 Description

The Guagnino water drainage plant is located in the municipality of Comacchio. Figure 23 shows the location and aerial framing of the site. The structure consists of two buildings: a side building of two floors above ground and adjacent to it the actual building constituting the plant. The central portion has horizontal dimensions of approximately 17.5 m x 11.80 m in width and a height of 9.60 meters. It consists of a main structure of solid clay masonry: behind the openings, the wall thickness varies from a maximum of 45 cm to a minimum of 29 cm (Figure 25). In the masonry there is a reinforced concrete frame, consisting of 12 pillars of 60x50 cm section and 50x50 cm section beams, which carries the bridge crane and a part of the weight of the roof. The frame was recently (2011) subject to retrofitting intervention using FRP reinforcements (shear and bending) that involved the bridge crane support beam. Above the frame the wall continues with a thickness of 28 cm up to the roof. The roof consists of a floor in brick and cement with two inclined pitches with a thickness of 16 + 4 cm; the horizontal thrust is absorbed by 9 rods, probably anchored to the correa beam. From an inspection of the plant and the documentation constituting the intervention with carbon fibers by the firm "Elletipi s.r.l.", it was found that there is no connection between the r.c. loom and the masonry.


*Figure 23. Localization and aerial photography of Guagnino pumping plant.*


*Figure 24. North side of the Guagnino pumping plant.*

*Figura 25. South side of the Guagnino pumping plant.*

The left portion (with reference to the photo in Figure 24) has horizontal dimensions 8 m x 11.80 and high at the intrados of the attic floor of 6.91 m; it consists of a load-bearing masonry structure with external perimeter walls of varying thickness at the openings (28-45 cm). There are two internal walls in the direction of the short side 30 cm thick. Inside the left portion there are two lifting groups, served by a small bridge crane that rests on two beams in c.a. supported by the masonry. Unlike the central portion there is an attic floor of 20 cm thickness. The right, eastward, side of the plant consists of a building of two storeys above ground of dimensions 7 m x 12.10 m. It is composed by a masonry structure consisting of external perimeter walls which, as for the left portion, are variable in thickness, while inside there is only one wall in the short direction of 30 cm thickness. From the structural point of view this building is completely separated from the rest of the construction by a joint. The storeys are made of 20 cm thick slabs.

## 3.2 Material properties

The typology of the Guagnino masonry is made of solid clay bricks and lime mortar. At a first visual examination during the inspection carried out at the plant, the mortar seemed to have good characteristics. Furthermore, the presence of a suitable transversal connection is assumed. Therefore, following the choices provided by the circular in tables C8.A.2.1 and C8.A.2.2 and assuming a knowledge level LC1, the following mechanical parameters of the masonry are obtained:

| | |
|---|---|
| Average compression strength | $f_m = 1.5 \cdot 1.3 \cdot 240 = 468\ [N/cm^2]$ |
| Average shear strength | $\tau_m = 1.5 \cdot 1.3 \cdot 6 = 11.7\ [N/cm^2]$ |
| Average Young modulus | $E = 1.5 \cdot 1500 = 2250\ [N/mm^2]$ |
| Average shear modulus | $G = 1.5 \cdot 500 = 750\ [N/mm^2]$ |
| Specific weight | $w = 18\ [kN/m^3]$ |

*Table 12. Mechanical parameters for masonry.*

The r.c. frame present within the central portion of the plant as a support for the bridge crane and part of the roof was investigated through a structural survey carried out by the company "Elletipi srl". The mechanical parameters obtained and deduced starting are reported below. The reinforcing bars have been assumed of the FeB38k type.

| | |
|---|---|
| Characteristic cubic compression strength | $R_{ck} = 19$ MPa |
| Characteristic cylindric compression strength | $f_{ck} = 16$ MPa |
| Average compression strength | $f_{cm} = 24$ MPa |
| Concrete Young modulus | $E_{cm} = 28607.90\ MPa$ |
| Steel characteristic yield strength | $f_{yk} = 375\ [N/mm^2]$ |
| Steel Young modulus | $E_s = 206'000\ [N/mm^2]$ |

*Table 13. Mechanical parameters for reinforced concrete.*

## 3.3 Loads

| **Roof** | | | |
|---|---|---|---|
| ***Material*** | ***□□□kN/m$^3$]*** | ***t [cm]*** | ***Weight [kN/m$^2$]*** |
| Slab (16+4 cm) | - | 16+4 | 1.90 |
| Impermeabilization | - | - | 0.10 |
| Tiles | - | - | 0.45 |
| Plaster | 20 | 1 | 0.20 |
| ***Total*** | | | ***2.65*** |
| ***$G_{1K}$*** | | | ***1.90*** |
| ***$G_{2K}$*** | | | ***0.75*** |
| ***Snow load $Q_{k1}$ : Comacchio(FE)*** | | | ***0.80*** |

*Table 14. Central building roof loads.*

| **Roof** | | | |
|---|---|---|---|
| ***Material*** | ***□□ [kN/m$^3$]*** | ***t [cm]*** | ***Weight [kN/m$^2$]*** |
| Slab (16+4 cm) | - | 16+4 | 1.90 |
| Impermeabilization | - | - | 0.10 |
| Tiles | - | - | 0.45 |
| ***Total*** | | | ***2.45*** |
| ***$G_{1K}$*** | | | ***1.90*** |
| ***$G_{2K}$*** | | | ***0.55*** |
| ***Snow load $Q_{k1}$ : Comacchio(FE)*** | | | ***0.80*** |

*Table 15. Left building roof load.*

| **First floor** | | | |
|---|---|---|---|
| **Material** | **□□□kN/m$^3$]** | **t [cm]** | **Weight [kN/m$^2$]** |
| Plaster | 20 | 1 | 0.20 |
| Slab 16+4 cm | - | - | 2.85 |
| Concrete | 17 | 3 | 0.51 |
| Pavement | - | - | 0.40 |
| Internal divisions | - | - | 1.00 |
| **Total** | | | **4.96** |
| **Structural load $G_1$** | | | **2.85** |
| **Non structural load $G_2$** | | | **2.11** |
| **Accidental load $Q_{k1}$** | | | **2.00** |
| **Secondo Solaio** | | | |
| ***Material*** | **□□□kN/m$^3$]** | **t [cm]** | **Weight [kN/m$^2$]** |
| Plaster | 20 | 1 | 0.20 |
| Slab 16+4 cm | - | - | 2.85 |
| **Total** | | | **3.05** |
| **Structural load $G_1$** | | | **2.85** |
| **Non structural load $G_2$** | | | **0.20** |
| **Accidental load $Q_{k1}$** | | | **0.50** |
| **Roof** | | | |
| | **□□□kN/m$^3$]** | **t [cm]** | **Weight [kN/m$^2$]** |
| Slab 16+4 cm | - | - | 2.85 |
| Impermeabilization | - | - | 0.10 |

| | | | |
|---|---|---|---|
| Tiles | - | - | 0.45 |
| **Total** | | | **3.40** |
| **Structural load $G_1$** | | | **2.85** |
| **Non structural load $G_2$** | | | **0.55** |
| **Snow load $Q_{k1}$ : Comacchio(FE)** | | | **0.80** |

*Table 16. Right building loads.*

## 3.4 Design seismic action

The design seismic action is defined through the horizontal psuedo-acceleration spectrum as per the current Italian Building Code presriptions.

- Nominal life: VN=50 years.
- Usage coefficient: Cu=1.5 .
- Limit State: SLV
- Soil category: D
- Topographic category: T1
- Behavior factor: q=1.

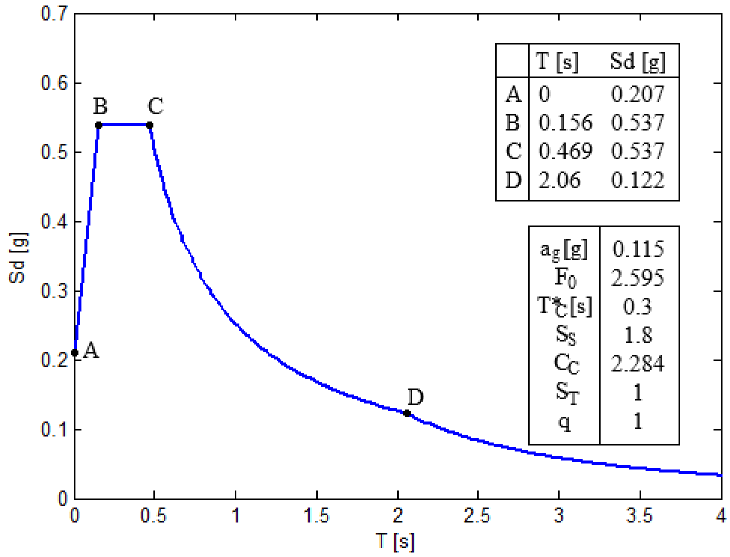



*Figure 26. Horizontal design pseudo-acceleration spectrum.*

## 3.5 Structural analyses

The effective load distribution due to the weight of the roof between the masonry and the frame was investigated: at first glance, it is not clear what the structural element taking on the weight of the roof is. An analysis on the FEM model showed that the vertical load is distributed as follows:
– 55% on masonry;
– 45% on the frame.

We then proceeded with the analysis of the structural elements, also keeping the three buildings that make up the construction separate.
As usual, preliminary analyzes were carried out on vertical loads (fundamental combination specified in paragraph 2.5.3 of the DM08). In addition to these, a verification of the tie-rod system on the central body was developed. Subsequently, the buildings' response to horizontal loads was analyzed.
With regard to the central body, consisting of only two parallel walls that support the roof, it was limited to local analyses of the first way: these were carried out using the Mc4Loc software with the addition of validations through manual calculations. On the side buildings instead, given the hypothesis of good overall behavior, non-linear global static (pushover) analyzes were developed using the 3Muri software, validated with linear static analyzes in an Excel environment.
The seismic masses were inserted according to the seismic combination specified in paragraph 2.5.3. of the NTC2008. Subsequently the earthquake was applied with the following 8 combinations:

- $\pm Ex \pm 0.3Ey$
- $\pm 0.3Ex \pm Ey$

Finally, the presence of continuous vertical lesions from the base to the top of the main walls has led to the idea of a differential settlement at the level of the foundation. Given the presence of deep cracks at the extrados of the bridge crane support beams (atypical position with respect to the stresses normally induced by the loads to which it is subjected, see Figure 31), the effect that a differential settlement in the foundation was also investigated can have on the frame.

# 3.6 Results

## 3.6.1 Vertical loads

The walls of the central body do not have uniform thickness along the whole height, and also in this case the slenderness was calculated with a thickness obtained by weighted average on the heights. In this way a slenderness equal to 20.4 was obtained, greater than the limit slenderness that the standard imposes for vertical loads. However we must take into account the fact that the thickness also varies along the width of the walls, so the wall was not assumed not verified a priori. The wall is not verified in the extreme case in which it takes on the full weight of the roof; if, on the other hand, only 55% of the load is attributed, the verification is satisfied.

On the building on the left there are some bearing walls characterized by excessive slenderness (equal to 25, due to a thickness of 28 cm for a height of 7 m). In this case, it was not possible to establish a safety index as the slenderness limit for vertical loads is largely exceeded.

As for the building on the right, the checks were carried out in the following two hypotheses:

- Bead as wide as the thickness of the wall;
- As wide as the semi-thickness of the wall.

In the first hypothesis the verification is less severe, however there are still some unverified masonry walls.

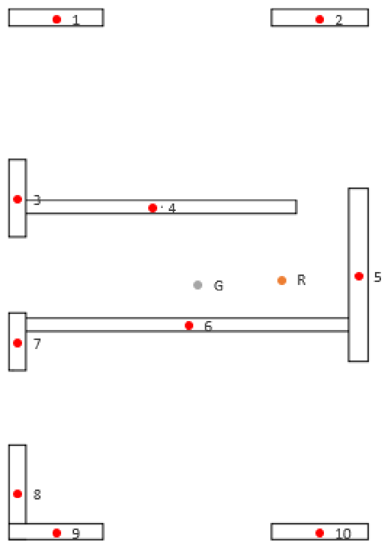



*Figure 27. Left building. Non-verified walls: 2,4,5,6,9,10.*

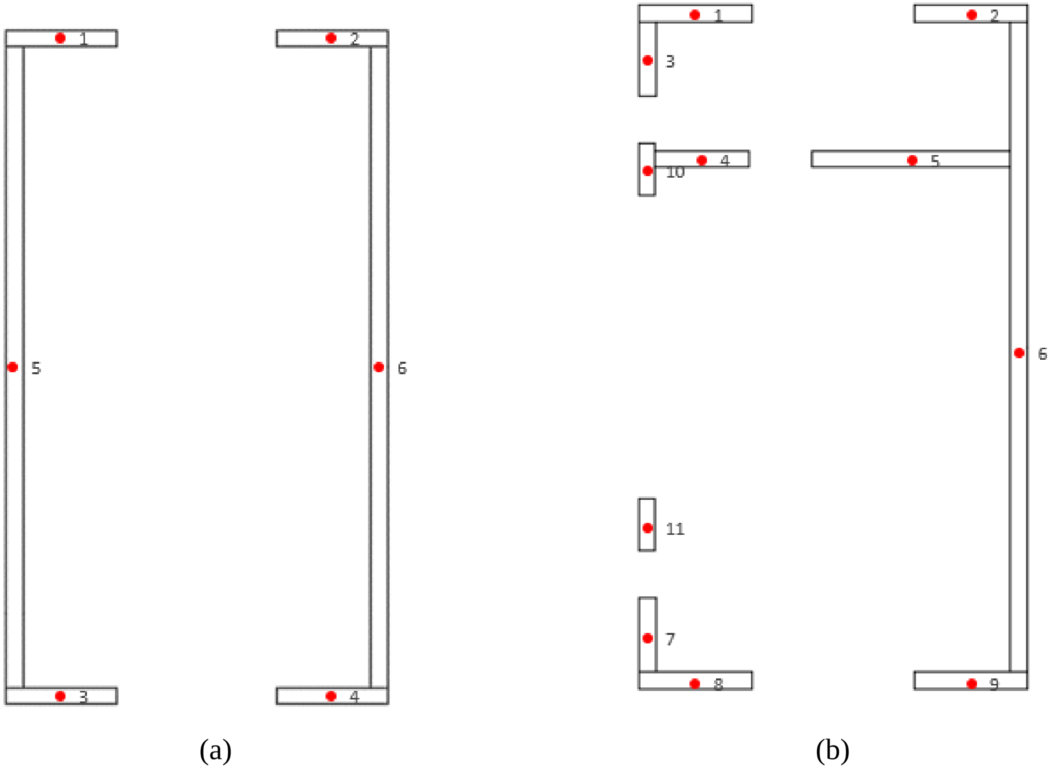



*Figure 28. Right building. (a) Ground floor, non-verified walls:10,11; (b) first floor, non-verified walls:5,6.*

## 3.6.2 Tie rods assessment

At a visual inspection, the tie rods present in the central body, placed to absorb the thrust of the two-pitched roof, seemed to have a small diameter in relation to their mutual distance of about 3 meters.

The calculation was carried out both by the Straus7 software and by manual calculation considering the static scheme shown in Figure 29. In the absence of a precise relief, different diameters were hypothesized; furthermore the calculation was developed both with vertical load factored according to the SLU combination of the current legislation and without factoring (the results are shown in Table 20). By applying a confidence factor of 1.35 to the resistance, and repeating the verification for various types of steels, we have obtained that to withstand the thrust determined by the current legislation, a steel tie-rod of at least S355 is required.

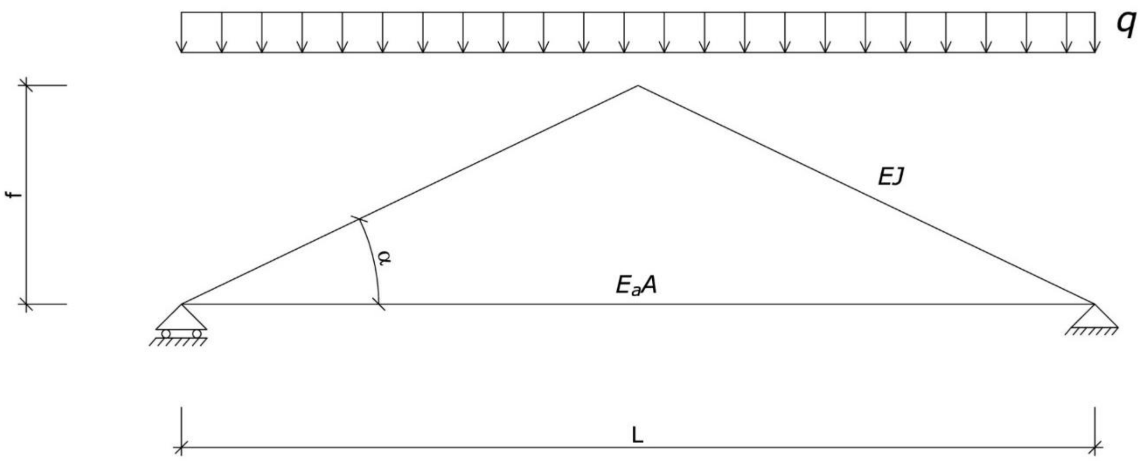



*Figure 29. Static scheme for the tie rod manual assessment.*

| | Thrust with load factorized SLU [kN] | Thrust without load factorized SLU [kN] |
|---|---|---|
| Ø 20 | 96.91 | 69.72 |
| Ø 22 | 98.44 | 70.83 |
| Ø 24 | 99.64 | 71.69 |

*Table 17. Computed thrust.*

## 3.6.3 Analyses of local mechanisms in the central building

The following mechanisms have been analyzed:

- Total overturning of the walls;
- Expulsion.

Table 21 shows the minor safety coefficients (capacity / demand) obtained with local analyzes. The worst situation was found with reference to the total overturning of the walls, for which there is a capacity / demand ratio corresponding to 33.2% (linear kinematic analysis procedure).

## 3.6.4 Global analyses on lateral buildings

In order to assess the seismic vulnerability of the plant a 3Muri model of each building is made. In each model all the walls that do not comply with the geometric requirements (Tab. 7.8.III DM08) and slenderness ($\lambda \leq 12$) that imposed by the standard in chap. 7 of the DM08. The 3Muri models are shown below.

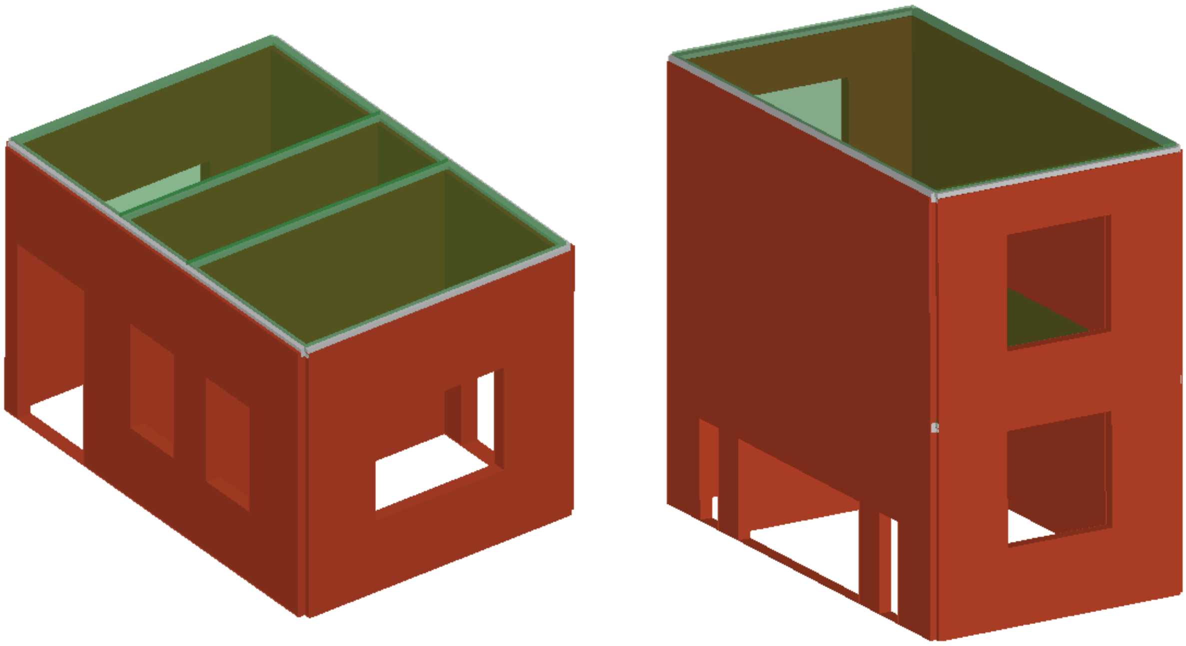


*Figure 30. 3Muri models of the right and left  building.*

The safety coefficients are shown in Table 12. The analyses carried out show the presence of a "weak direction" for both buildings, ie a direction in which there is not enough masonry to withstand the seismic action. This result is also validated by a check carried out according to the limits reported in Table 7.8.III of the DM08. This control was made as a simple term of comparison, as these limits refer to "simple" structures, a category in which the buildings do not fit.

### 3.6.5 Summary

The following is a summary of the results obtained regarding the seismic vulnerability analysis of buildings. In addition to the safety coefficients (column αRd / αEd) the trip time (column TI) was also reported.

| | Description | $a_{Rd}$ $[m/s^2]$ | $a_{Ed}$ $[m/s^2]$ | $\frac{a_{Rd}}{a_{Ed}}$ | $TR_{Rd}$ $[anni]$ | TI $[anni]$ |
|---|---|---|---|---|---|---|
| **Local mechanisms (central building)** | Overturning | 0.339 | 1.02 | 33.2 % | 42 | 3 |
| | Expulsion | 0.756 | 1.02 | 74.1 % | 319 | 22 |
| **Global analyses (Lateral buildings)** | Left building | 1.71 | 2.02 | 84.7 % | 437 | 31 |
| | Right building | 1.26 | 2.02 | 62.4 % | 204 | 14 |

*Table 18. Summary of the analyses.*

### 3.6.6 Analysis of the r.c. frame

For what concerns the damage status of the bridge crane support beams (Figure 31), it was thought that it could be due to a constraining failure of one of the side portions of the system: this could have caused a redistribution of the stresses in the frame such as to stress traction the beams for use of the bridge crane.
The FEM analysis with overlapping of the yielding-vertical load effects has effectively shown, in the beams near the settlement, the presence of traction stresses concomitant with values of zero moment at the center line. The traction would therefore have caused the cracking of the concrete to the extrados of the beam (as the stresses were absorbed by the FRP reinforcement at the intrados). A resistance check on the section confirmed the hypotheses made.

*Figure 31. Damage state of the r.c. beams.*

## 3.7 Suggested retrofit interventions

### 3.7.1 Left building

In order to limit the excessive slenderness of the walls one can think of installing a metal curb at the height of the extrados of the openings, such that it prevents the masonry from going into crisis out of plane due to vertical loads. The metal curb can be made, as a first approximation, using a UPN120 type profile and anchored with masonry dowels in the order of 3-4 per meter. In Figure 32 we can see a cutaway of the left portion with the positioning of the metal curb. The installation of the metal curb allows a clear improvement in the outcome of the checks relating to the lower portion of the masonry, effectively eliminating the vulnerability to vertical loads.

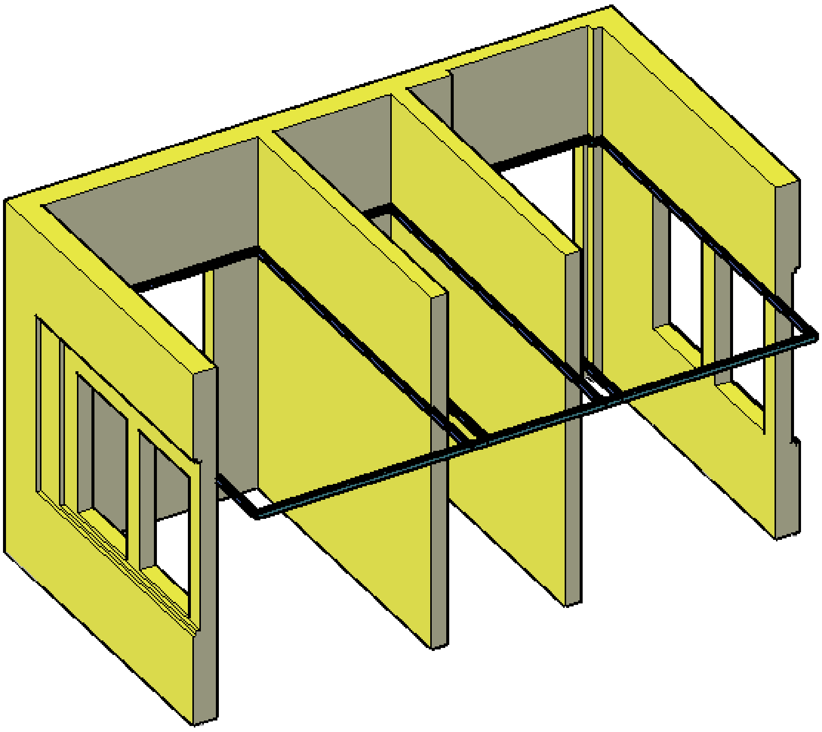


*Figure 32. Reftrofit of the left building.*

## 3.7.2 Central building

As seen for the S. Antonino plant, a possible intervention to avoid overturning phenomena is to install a new line of tie rods. These would be inserted just below the correa beam that binds the existing tie rods, as long as they do not interfere with the operation of the bridge crane. In order to carry out the necessary checks, tie-rods in S235 steel with a diameter of mm 20 mm were assumed, on a square plate of 50 × 50 cm. With 5 tie rods, the overturning check is satisfied.

## 3.8 Conclusions

As for the Guagnino di Comacchio plant, the greatest urgency is given by the vulnerability to vertical loads of the walls of the building on the left. On the other hand, the horizontal situation is similar to that of the Sant'Antonino plant: the local overturning mechanism of the portions of masonry making up the central portion of the plant is the most vulnerable with a safety index of 33% and a time of corresponding intervention equal to 3 years. However, in the hypothesis of inserting a new line of tie-rods it is possible to remedy this deficiency.

Therefore at present the Guagnino plant, in relation to the hypotheses carried out, does not appear to be verified at the vertical loads the walls having an excessive slenderness in the building on the left and the local mechanisms the walls in the central portion. In the hypothesis of remedying it through the proposed interventions, we can assume as the intervention time the one related to the worst mechanism of II mode, that is to say the global analysis of the right portion or building of the plant that has highlighted a time of 14-year intervention.

# CONCLUSIONS

The analyses carried out on the first three drainage systems have shown a strong vulnerability to horizontal actions, and in some cases also to vertical actions.

The purpose of this report was to evaluate the seismic vulnerability of these buildings, however the study cannot ignore any deficiencies in vertical loads, especially if the building is used for residential purposes. Therefore, it is advisable to intervene first of all on those buildings that have highlighted problems of this kind.

Shifting attention to horizontal actions, the most dangerous situation corresponds to the Valle Lepri Acque Alte plant. The presence of squat pillars at the base of the building, having a weak section corresponding to the position in which there are maximum stresses, is absolutely inadequate even in areas characterized by low seismicity.

The other two plants are fairly similar conditions, in which the greatest vulnerability is given by the overturning of the walls of the central bodies. We recommend at least the application of new tie rods to avoid this type of mechanism.

It should be remembered that these analyzes have been developed without precise observations on the conditions of materials and construction details. A series of investigations aimed at determining the current state of concrete, reinforcing steel, masonry, clamping between orthogonal walls, and the effective presence of reinforced concrete curbs would therefore be appropriate. In the absence of more precise data, the safety indexes reported in the report refer to the simplification hypotheses carried out.